Form 36

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Interested committees:

Title: Draft BS EN 1998-1-2 Eurocode 8 - Design of structures for earthquake resistance. Part 1-2. Buildings.

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Introduction

This draft standard is based on national and international discussions. Your comments on this draft are invited and will assist in the preparation of the consequent standard.

For international standards, comments will be reviewed by the relevant UK national committee before sending the consensus UK vote and comments to the international committee, which will then decide appropriate action. If the international standard is approved, it is usual for the text to be published as a British Standard.

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Date: xx/xx/20xx	Document: ISO/DIS xxxx

1	2	(3)	4	5	(6)	7
MB	Clause No./ Subclause No./Annex **(e.g. 3.1)**	Paragraph/ Figure/Table/Note	**Type of comment**	**Commend (justification for change) by the MB**	**Proposed change by the MB**	**Secretariat observations on each comment submitted**
	3.1	Definition 1	ed	Definition is ambiguous and needs clarifying.	Amend to read '...so that the mains connector to which no connection...'	
	6.4	Paragraph 2	te	The use of the UV photometer as an alternative cannot be supported as serious problems have been encountered in its use in the UK.	Delete reference to UV photometer.	

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EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

DRAFT
prEN 1998-1-2

September 2023

ICS 91.120.25

Will supersede EN 1998-1:2004

English Version

Eurocode 8 - Design of structures for earthquake resistance - Part 1-2: Buildings

Eurocode 8 - Calcul des structures pour leur résistance au séisme - Part 1-2: Bâtiments

Eurocode 8 - Auslegung von Bauwerken gegen Erdbeben - Teil 1-2: Hochbauten

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European foreword

This document (prEN 1998-1-2:2023) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document is currently submitted to the CEN Enquiry.

This document will supersede part of EN 1998-1:2004.

The first generation of EN Eurocodes was published between 2002 and 2007. This document forms part of the second generation of the Eurocodes, which have been prepared under Mandate M/515 issued to CEN by the European Commission and the European Free Trade Association.

The Eurocodes have been drafted to be used in conjunction with relevant execution, material, product and test standards, and to identify requirements for execution, materials, products and testing that are relied upon by the Eurocodes.

The Eurocodes recognize the responsibility of each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level through the use of National Annexes.

A list of the main changes compared to the previous edition will be added at Formal Vote.

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0 Introduction

0.1 Introduction to the Eurocodes

The Structural Eurocodes comprise the following standards generally consisting of a number of parts:

— EN 1990 Eurocode: Basis of structural and geotechnical design

— EN 1991 Eurocode 1: Actions on structures

— EN 1992 Eurocode 2: Design of concrete structures

— EN 1993 Eurocode 3: Design of steel structures

— EN 1994 Eurocode 4: Design of composite steel and concrete structures

— EN 1995 Eurocode 5: Design of timber structures

— EN 1996 Eurocode 6: Design of masonry structures

— EN 1997 Eurocode 7: Geotechnical design

— EN 1998 Eurocode 8: Design of structures for earthquake resistance

— EN 1999 Eurocode 9: Design of aluminium structures

— New parts are under development, e.g. Eurocode for design of structural glass

The Eurocodes are intended for use by designers, clients, manufacturers, constructors, relevant authorities (in exercising their duties in accordance with national or international regulations), educators, software developers and committees drafting standards for related product, testing and execution standards.

NOTE Some aspects of design are most appropriately specified by relevant authorities or, where not specified, can be agreed on a project-specific basis between relevant parties such as designers and clients. The Eurocodes identify such aspects making explicit reference to relevant authorities and relevant parties.

0.2 Introduction to EN 1998 (all parts)

EN 1998 defines the rules for the seismic design of new buildings and engineering works and the assessment and retrofit of existing ones, including geotechnical aspects, as well as temporary structures.

NOTE This standard also covers the verification of structures in the seismic situation during construction, when required.

Attention should be paid to the fact that, for the design of structures in seismic regions, the provisions of EN 1998 should be applied in addition to the relevant provisions of EN 1990 to EN 1997 and EN 1999. In particular, EN 1998 should be applied to structures of consequence classes CC1, CC2 and CC3, as defined in EN 1990:2023, 4.3. Structures of consequence class CC4 are not fully covered by the Eurocodes but may be required to follow EN 1998, or parts of it, by the relevant Authorities.

By nature, perfect protection (a null seismic risk) against earthquakes is not feasible in practice, namely because the knowledge of the hazard itself is characterized by a significant uncertainty. Therefore, in Eurocode 8, the seismic action is represented in a conventional form, proportional in amplitude to earthquakes likely to occur at a given location and representative of their frequency content. This representation is not the prediction of a particular seismic movement, and such a movement could give rise to more severe effects than those of the seismic action considered, inflicting damage greater than the one described by the Limit States contemplated in this standard.

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Not only the seismic action cannot be predicted, but in addition, it should be recognized that engineering methods are not perfectly predictive when considering the effects of this specific action, under which structures are assumed to respond in the non-linear regime. Such uncertainties are taken into account according to the general framework of EN 1990, with a residual risk of underestimation of their effects.

EN 1998 is subdivided in various parts:

— EN 1998-1-1, *Eurocode 8 — Design of structures for earthquake resistance – Part 1-1: General rules and seismic action;*

— EN 1998-1-2, *Eurocode 8 — Design of structures for earthquake resistance – Part 1-2: Buildings;*

— EN 1998-2, *Eurocode 8 — Design of structures for earthquake resistance – Part 2: Bridges;*

— EN 1998-3, *Eurocode 8 — Design of structures for earthquake resistance – Part 3: Assessment and retrofitting of buildings and bridges;*

— EN 1998-4, *Eurocode 8 — Design of structures for earthquake resistance – Part 4: Silos, tanks, pipelines, towers, masts and chimneys;*

— EN 1998-5, *Eurocode 8 — Design of structures for earthquake resistance – Part 5: Geotechnical aspects, foundations, retaining and underground structures.*

0.3 Introduction to prEN 1998-1-2

prEN 1998-1-2 provides specific requirements for earthquake resistant design of new buildings, including rules for structural materials, additional to the ones in other Eurocodes. This document contains, in its Clause 14 related to masonry buildings, specific provisions that simplify the design of "simple masonry buildings". This document also contains provisions for the design of base-isolated buildings and buildings with energy dissipation systems.

prEN 1998-1-2 is subdivided in fifteen clauses and includes thirteen annexes, where Annexes A, B, H, J, K and M are informative and Annexes C, D, E, F, G, I and L are normative.

0.4 Verbal forms used in the Eurocodes

The verb "shall" expresses a requirement strictly to be followed and from which no deviation is permitted in order to comply with the Eurocodes.

The verb "should" expresses a highly recommended choice or course of action. Subject to national regulation and/or any relevant contractual provisions, alternative approaches could be used/adopted where technically justified.

The verb "may" expresses a course of action permissible within the limits of the Eurocodes.

The verb "can" expresses possibility and capability; it is used for statements of fact and clarification of concepts.

0.5 National Annex for prEN 1998-1-2

National choice is allowed in this standard where explicitly stated within notes. National choice includes the selection of values for Nationally Determined Parameters (NDPs).

The national standard implementing prEN 1998-1-2 can have a National Annex containing all national choices to be used for the design of buildings to be constructed in the relevant country.

When no national choice is given, the default choice given in this standard is to be used.

When no national choice is made and no default is given in this document, the choice can be specified by a relevant authority or, where not specified, agreed for a specific project by the relevant parties.

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National choice is allowed in prEN 1998-1-2 through notes to the following clauses:

4.1(2)	4.1(3)	4.2(1) NOTE 1	4.2(1) NOTE 2
4.2(1) NOTE 3	6.3.2.1(2)	7.2.2(1)	10.3.4(1)
10.4.5(3)	11.6.3(4)	13.2.2(2)	14.3(5)
14.6.1.3(2)	14.6.1.4(3)	14.8.2(6)	14.8.3(2) NOTE 1
14.8.3(2) NOTE 2	L.3(2)e)	L.3(2)f)	

National choice is also allowed in prEN 1998-1-2 on the application of the following informative annexes:

Annex A	Annex B	Annex H	Annex J
Annex K	Annex M		

The National Annex can contain, directly or by reference, non-contradictory complementary information for ease of implementation, provided it does not alter any provisions of the Eurocodes.

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1 Scope

1.1 Scope of prEN 1998-1-2

(1) This document is applicable to the design and verification of new buildings and temporary structures in seismic regions. It gives specific rules for the design and verification relevant to buildings of consequence classes CC1, CC2 and CC3, as defined in EN 1990:2023, 4.3.

NOTE The assessment and retrofitting of existing buildings is covered in prEN 1998-3.

(2) Unless specifically stated, prEN 1998-1-1 and prEN 1998-5 apply.

(3) EN 1998-1-2 is applicable in complement to the other relevant Eurocodes.

NOTE This document contains only those provisions that, in addition to the provisions of the other relevant Eurocodes, are used for the design of new buildings and temporary structures in seismic regions. prEN 1998-1-2 complements in this respect the other Eurocodes.

1.2 Assumptions

(1) The assumptions of prEN 1998-1-1:2022, 1.2 are assumed to be applied.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

NOTE See the Bibliography for a list of other documents cited that are not normative references, including those referenced as recommendations (i.e. in 'should' clauses), permissions ('may' clauses), possibilities ('can' clauses), and in notes.

EN 1990:2023, *Eurocode — Basis of structural and geotechnical design*

prEN 1995-1-1:2023, *Eurocode 5 — Design of timber structures — Part 1-1: General rules and rules for buildings*

prEN 1998-1-1:2022, *Eurocode 8 — Design of structures for earthquake resistance — Part 1-1: General rules and seismic action*

prEN 1998-5:2022, *Eurocode 8 — Design of structures for earthquake resistance — Part 5: Geotechnical aspects, foundations, retaining and underground structures*

EN 12512, *Timber structures — Test methods — Cyclic testing of joints made with mechanical fasteners*

ISO 80000 (all parts), *Quantities and units*

3 Terms, definitions and symbols

3.1 Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1990, prEN 1998-1-1:2022, 3.1, and the following apply.

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3.1.1

balloon frame construction

construction type that uses long continuous framing members running from the foundation or base of the timber structure to the top of the building, with intermediate floor structures nailed to them. The walls are therefore continuous vertically

3.1.2

bounding frame

columns and upper and lower beams that surround infill walls and provide structural support

3.1.3

box behaviour

a structure behaves as a box if it is composed of stiff vertical and horizontal planes connected to each other along lines where there are no relative displacements between the planes

3.1.4

carpentry connection

connection where loads are transferred primarily in compression by shaping the connecting timber elements. Fasteners can be present but only to provide integrity to the connection

Note 1 to entry: Skew notch, mortice and tenon, and halving connections are examples of carpentry connections.

3.1.5

cladding

element that covers the surface of a building. Claddings are fixed by means of connections onto the frame and protects the building from e.g. humidity, light, temperature, wind. Claddings can be made of different products like precast concrete panels, lightweight steel panels, glazing

3.1.6

coupling effect

refers to the increase of horizontal stiffness and strength of the building that results from the flexural stiffness of floors, ring beams or spandrels and the moment frame interaction of these elements with walls. The coupling effect leads to a change in internal forces (axial load, shear force and bending moment) in vertical elements that results from shear forces and bending moments transferred by the horizontal members

3.1.7

critical region

region of a primary seismic element, where the most adverse combination of action effects (M, N, V, T) occurs and where plastic hinges can form

Note 1 to entry: In concrete buildings, critical regions are potential dissipative zones. The length of the critical region is defined for each type of primary seismic element in the relevant subclauses of Clause 10.

3.1.8

diaphragm

horizontal or nearly horizontal structural member, such as a floor or roof system, used to transfer inertial lateral forces to vertical members of the seismic action resisting system

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3.1.9

diaphragm chord

boundary component perpendicular to the inertial force that is provided to resist tension or compression caused by the diaphragm moment coupling effect

3.1.10

diaphragm collector

component parallel to the inertial force that transfers lateral forces from the diaphragm of the structure to vertical members of the seismic action resisting system

3.1.11

dowel-type connection

connection with dowel-type mechanical fasteners loaded perpendicularly to their axis

Note 1 to entry: Nails, staples, screws, dowels and bolts are examples of dowel-type mechanical fasteners.

3.1.12

ductility

ratio between the ultimate deformation of a member or connection and its deformation at the elastic limit

3.1.13

infill

masonry wall constructed within the plane of an internal or external structural frame and bounded by that frame

3.1.14

integrated connection systems

couple structure and cladding panels response. If the supporting structure is a moment frame and if the cladding panels are elements of walls, the integrated structural system can be a moment frame-equivalent dual system or a wall-equivalent dual system, depending on the performance of the connections and of the relative stiffness of the claddings and of the supporting frame

3.1.15

interacting cladding

cladding activated by deflection of the supporting frame and significantly affecting the structural response of a building

3.1.16

interacting infill

infill activated by deflection of the bounding frame and significantly affecting the structural response of a building

3.1.17

isolation interface

surface or zone which separates the substructure and the superstructure and where the isolation system is located

Note 1 to entry: Arrangement of the isolation interface at the base of the structure is usual in buildings, tanks and silos. In bridges, the isolation system is combined with the bearings and the isolation interface lies between the deck and the piers or abutments.

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3.1.18

isostatic connection system

connection system which allows a free relative displacement between cladding and supporting structure under seismic conditions

3.1.19

non-interacting infill

infill designed not to significantly change the structural response of a building, e.g. infill separated from the frame by a construction joint

3.1.20

partition

wall or division used to divide the internal space into separate rooms. Partitions can be made of different materials (e.g. bricks, glass or other). Partition walls are placed inside the building between the floor above and the floor under it and vertical elements like walls or columns. A partition is not meant to play any structural role other than supporting itself

3.1.21

pier

part of a masonry wall between vertical limits, which can be a wall free edge, a window, a door or a stiffening wall. The length of a pier cross section is the distance between two such interruptions and its height is that of the smaller adjacent opening

3.1.22

platform frame construction

timber construction type that involves floor structures bearing onto wall panels, thereby creating a 'platform' for construction of the next level of wall panels. The timber walls are therefore interrupted by the floor structures

3.1.23

principal direction of a building

horizontal direction in which a main lateral force resisting system is designed

3.1.24

primary seismic member

structural member which participates to the lateral load resisting system

3.1.25

primary structure

set of primary seismic members over the building

3.1.26

primary bracing

set of primary members installed in a vertical plane at a given level of the building

3.1.27

rigid joints

joints with negligible flexibility

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3.1.28
ring beams and ring ties
horizontal elements that connect all masonry piers at a specific elevation of a building

3.1.29
secondary seismic member
structural member which does not participate to the lateral load resisting system, but keeps its other functions, like carrying gravity loads, during the seismic event

3.1.30
semi-rigid joints
joints with significant flexibility

3.1.31
spandrel
masonry element with a horizontal axis spanning between two adjacent piers

3.1.32
substructure
part of the structure, which is located under the isolation interface, including the foundation

Note 1 to entry: The lateral flexibility of the substructure(s) is generally negligible in comparison to that of the isolation system, but this is not always the case.

3.1.33
superstructure
part of the structure that is located above the isolation interface

3.1.34
transfer zone

zone of a building where vertical components of the primary structure are interrupted for architectural reasons (for open spaces, car parks or setbacks, etc.)

3.1.35
walls
vertical members with an elongated cross section; in reinforced concrete walls, this means a length to thickness ratio of the cross section greater than 4

3.2 Symbols and abbreviations

For the purposes of this document, the symbols and abbreviations given in EN 1990:2023, 3.2 and in prEN 1998-1-1:2022, 3.2 apply.

For the symbols related to materials, as well as for symbols not specifically related to the seismic design situation, the provisions of the relevant Eurocodes apply.

In addition, further symbols and abbreviations, used in connection with the seismic design situation, are defined in the present standard where they occur, for ease of use. However, the most frequently occurring symbols used in prEN 1998-1-2 are listed and defined in 3.2.1 and additional abbreviations are given in 3.2.2.

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3.2.1 Symbols

3.2.1.1 Upper case Latin symbols

A_{br}	area of a brace section
A_c	column cross-section area
$A_{j,ef}$	effective cross-sectional area of the beam-column joint
$A_{mp,i}$	area of the section of primary pier *i*
$A_{mp,tot}$	total pier section area
A_p	pier section area
$A_{s,int}$	integrity reinforcement area in the flat slab
A_{s1}	area of the beam's top longitudinal reinforcement used to calculate the design shear in the joint
A_{s2}	area of the beam's bottom longitudinal reinforcement used to calculate the design shear in the joint
A_{sL}	area of the longitudinal bars
A_{st}	area of one leg of the transverse reinforcement within the lap zone
A_{stif}	area of a stiffener
A_{sw}	area of shear reinforcement
A^+ (A^-)	areas of the vertical projections of the cross sections of a tension diagonal
B_i	one dimension of a rectangular floor together with L_i
$B_{CLT,i,j}$	length of the j^{th} CLT shear wall at the i^{th} storey
$C_{eff,j}$	effective damping coefficient of a velocity-dependent energy dissipation device
CF_1	weighting factor of the response at the stage of maximum displacement
CF_2	weighting factor of the response at the stage of maximum velocity
C_j	damping coefficient of the velocity-dependent energy dissipation device *j*
CVF	correction factor for velocity
D_{lower}	smallest sieve size of the coarsest fraction of the aggregates permitted by the specification of the concrete
D_{max}	maximum sieve size of the coarsest fraction of the aggregates
E_a	modulus of elasticity of structural steel
E_{cm}	mean modulus of elasticity of concrete
E_d	design value of the action effect in the seismic design situation, including possible influence of second-order effects
$E_{des,k}$	elastic strain energy stored by the energy dissipation devices at *k*-th storey while the primary seismic elements of the main structural system at the *k*-th storey remain in the elastic domain

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$E_{dH,k}$ energy dissipated through plastic deformation by the energy dissipation devices at the k-th storey while the primary seismic elements of the main structural system at the k-th storey remain in the elastic domain

$E_{dH,k,max}$ maximum energy dissipation demand at the k-th storey from the energy dissipation devices under the reference seismic action

$E_{dH,k,SD}$ amount of energy that can be dissipated by the energy dissipation devices at the k-th storey under cyclic deformations when any energy dissipation device on the storey attains its SD limit state

E_{di} design value of the action effect on the zone or element i in the seismic design situation

$E_{d,E}$ combination of action effects $E_{F,E}$ and $E_{F,G}$

E_e amount of energy that the structure can absorb while the primary seismic elements of the main structural system remain in the elastic domain

$E_{F,E}$ action effect from the analysis of the design seismic action

$E_{F,G}$ action effect due to the non-seismic actions included in the combination of actions in the seismic design situation

$E_{H,k}$ required amount of energy dissipation through plastic deformation at the k-th storey of the building

E_{oop} resultant horizontal force acting on a wall and which creates out-of-plane deformation of the wall

$E_{pes,k}$ elastic strain energy stored by the primary seismic elements of the main structural system at the k-th storey while these elements remain in the elastic domain

$E_{pH,k}$ energy dissipated by the primary seismic elements of the main system at the k-th storey of the building

$E_{pH,k,max}$ maximum energy dissipation demand at the k-th storey in the primary seismic elements of the main structural system under the design earthquake

$E_{pH,k,SD}$ amount of energy that can be dissipated by the primary seismic elements of the main structural system at the k-th storey under cyclic deformations before reaching the SD limit state

E_s design value of the modulus of elasticity of reinforcing steel

E_{sec} short-term secant modulus of elasticity of masonry

F^* force of the equivalent SDOF system

F_1 load at the first cycle and at a certain deformation δ in a cyclic test performed in accordance with EN 12512

F_3 load at the third cycle and at a certain deformation δ in a cyclic test performed in accordance with EN 12512

F_{ap} horizontal seismic force, acting at the centre of mass of the ancillary element in the most unfavourable direction

F_b seismic base shear force

F_c compression force

$F_{b,Rd}$ design bearing resistance of the member-to-sheathing connection

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F_{Ed} design value of the lateral force acting on a steel shear wall in the seismic design situation

$F_{Ed,E}$ action effect (shear force, bending moment, or axial force) in the non-dissipative joint, connection, or member, due to the design seismic action

$F_{Ed,E,s}$ action effect in the shear connections due to the design seismic action

$F_{Ed,G}$ action effect (shear force, bending moment, or axial force) in the non-dissipative joint, connection, or member, due to the non-seismic actions in the seismic design situation

F_H hysteresis loop adjustment factor

F_i horizontal force acting on storey *i*

F_{i1} design lateral force at *i*-th floor in the first mode

F_{im} design lateral force at *i*-th floor in the *m*-th mode

$F_{M,Ed}$ design shear force in a fastener in the outer circle due to the bending moment transmitted by the joint

F_{max} maximum load attained on the 1st cyclic envelope load-deformation curve in a cyclic test performed in accordance with EN 12512

F_{oop} resultant horizontal force acting on an out-of-plane loaded pier

$F_{Rd,b}$ design value of the strength of the non-dissipative, brittle, components

$F_{Rd,hd,i,j}$ tensile-vertical design strength of the anchoring connections against overturning of the j^{th} CLT shear-wall at the i^{th} storey

$F_{Rk,b}$ characteristic value of the strength of the non-dissipative, brittle, components

$F_{Rd,c}$ design strength in seismic conditions of the single timber-to-timber connection used in the vertical joint

$F_{Rd,d}$ design value of the strength of the dissipative, ductile, zones

$F_{Rk,d}$ characteristic value of the strength of the dissipative, ductile, zones

$F_{Rd,hd}$ design strength in seismic conditions of the anchoring connection against overturning

$F_{Rd,s}$ design strength of the shear connections

$F_{Rm,d,M}$ mean maximum resistance of the dissipative connection under seismic loading

$F_{Rm,d,U}$ mean ultimate resistance of the dissipative connection under seismic loading

$F_{Rm,d,Y}$ mean yield resistance of the dissipative connection under seismic loading

$F_{t,90,Ed}$ design tensile force in reinforcement

F_u ultimate load attained on the 1st cyclic envelope load-deformation curve corresponding to the ultimate deformation δ_u in a cyclic test performed in accordance with EN 12512

$F_{v,Rd}$ design shear resistance of screws

$F_{v,Rk,d}$ characteristic strength of the selected ductile failure mode providing energy dissipation, according to prEN 1995-1-1

$F_{v,Rk,nd}$ characteristic strength of the less ductile failure mode

F_y yield load of a dissipative timber connection

H_b height of the building (in m) above the foundation or the top of a rigid basement

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I_a	moment of inertia of the structural steel cross-section with respect to the axis through the centroid of the transformed section
I_b	second moment of area of the beam section
I_c	moment of inertia of the uncracked concrete section with respect to the axis through the centroid of the transformed section
I_{oop}	moment of inertia of a pier about the out-of-plane bending axis
I_s	moment of inertia of the steel reinforcement with respect to the axis through the centroid of the transformed section
$I_{x,i}$, $I_{y,i}$	mass moments of inertia of the floor around axis *x* and *y* respectively, going through the floor centre of mass G_i
K_{eff}	effective stiffness
$K_{EDD,sj}$	horizontal elastic storage stiffness component of the viscoelastic energy dissipation device *j* installed at the *s*-th storey
$K_{SLS,v,mean}$	slip modulus of a connection at serviceability limit state calculated according to prEN 1995-1-1:2023, 5.10.1
$K_{SLS,v,mean,c}$	slip modulus of a connection measured in cyclic tests according to EN 12512
$K_{SLS,anc}$	stiffness of anchoring 2D- or 3D-connector against overturning
$K_{SLS,con}$	stiffness of the single timber-to-timber connection used in the vertical joint
L_{br}	brace length
L	distance between parallel walls
L_e	distance between the two-outermost lateral load resisting elements, measured perpendicularly to the direction of the seismic action considered
L_b	beam length
L_h	distance between dissipative zones at ends of a beam span
L_i	floor dimension perpendicular to the direction of the seismic action
L_{st}	distance between lateral-torsional restraints
$M_{b,Rd}$	design bending resistances of a steel beam cross-section
$M_{dst,d,E}$	overturning moment due to design seismic action
M'_{Edw}	design bending moment of the wall from the analysis for the seismic design situation
$M'_{Edw,base}$	design bending moment at the base of the wall from the analysis for the seismic design situation
M_{Ed}	design bending moment from the analysis for the seismic design situation
$M_{Ed,E}$	design rocking moment of the shear wall due to the seismic action
$M_{Ed,E,i,j}$	design rocking moment of the j^{th} segmented shear wall at the i^{th} storey due to the seismic action
M_{Edw}	design bending moment of the wall in the seismic design situation (envelope)
$M_{i,d}$	moment at beam or column end *i*

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$M_{T,j,Ed}$	design moment in a joint *j*
$M_{j,Rd}$	resistance of continuous beam-to column connections
$M_{oop,Ed}$	design bending moment due to horizontal forces generating out-of-plane deformation
$M_{oop,Rd}$	design bending resistance against out-of-plane action effects
M_{pier}	total mass of a pier
$M_{p,link}$	design value of plastic moment resistance of a link in eccentrically braced frames
$M_{pl,Rd}$	design value of plastic moment resistance of a steel component
$M_{Rd,b}$	design moment of resistance of a beam
$M_{Rd,b,i}$	design moment of resistance of a beam at end *i*
$M_{Rd,c}$	design moment of resistance of a column
$M_{Rd,c,i}$	design moment of resistance of a column at end *i*
$M_{Rd,inf}$	design moment of resistance at the bottom of a column
$M_{Rd,rock}$	design rocking strength of the shear wall
$M_{Rd,rock,i,j}$	design rocking strength of the j^{th} segmented shear wall at the i^{th} storey including the stabilizing effect of the vertical load
$M_{Rdw,base}$	design moment of resistance at the base of the wall
$M_{stb,d,G}$	stabilizing moment due to gravity loads in the seismic design situation
N	axial force
$N_{Rd,b}$	design axial resistance of a beam cross-section (squash load, buckling load)
$N_{Rd,c}$	design resistance of a column cross-section
$N_{C,j,Ed}$	design compression in a joint *j*
$N_{CT,BRS}$	Eulerian critical load of the sleeve of a buckling restrained brace
N_{Ed}	design axial force from the analysis for the seismic design situation
M_{Edcb}	moment in the coupling beam from the analysis
$N_{Ed,E}$	axial force due to the design seismic action
$N_{Ed,G}$	axial force due to the non-seismic actions in the seismic design situation
$N_{Ed,i,j}$	total compressive axial load acting on the j^{th} CLT shear wall at the i^{th} storey
$N_{EDD,j1}$	maximum axial force in energy dissipation device *j* in the first mode
$N_{EDDd,j1}$	axial viscous damping force component in energy dissipation device *j* in the first mode
$N_{EDDe,1j}$	axial elastic force component in a viscoelastic-type of energy dissipation device *j* in the first mode
$N_{f,Ed}$	axial force in a compressed flange
N_i	number of shear walls parallel to the seismic action at the i^{th} storey
N_{max}	maximum number of storeys in simple masonry buildings

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$N_{pl,Rd}$ design value of yield resistance in tension of the gross cross-section of a member in accordance with EN 1993-1-1

N_s number of storeys of a building

$N_{T,j,Ed}$ design tension in a joint *j*

$N_{u,link}$ upper design resistance of a link in an eccentric bracing

P_{tot} total gravity load at and above a storey in the seismic design situation

Q_m local forces

$R_{c,Rd}$ in-plane lateral resistance corresponding to the resistance of the member to sheathing connection in a light weight steel structure

R_d design resistance of a connection as given in the reference Eurocode

R_{di} design resistance of the zone or element *i*

R_{fy} plastic resistance of a dissipative member

R_{fo} plastic resistance of an aluminium dissipative member

S_a value of the floor acceleration spectrum

$S_d(T_1)$ ordinate of the reduced spectrum at period T_1

S_e ordinate of the elastic response spectrum

S_{ea} value of S_e at the ancillary element period

$S_{ep,1}$ value of S_e at the period of the first (fundamental) mode of the primary structure

$S_{ep,i}$ value of S_e at the period of the i^{th} mode of the primary structure

S_δ seismic action index

T_1 fundamental period of the structure with an energy dissipation system in elastic conditions or fundamental period of vibration of the building for lateral motion in the direction considered

$T_{1,est}$ estimate of the fundamental period of vibration of the building

T_A short period cut-off of the zero-period spectral acceleration – see prEN 1998-1-1:2022, 5.2.2.2(1)

T_{an} natural period of the ancillary element

T_C upper corner period of vibration of the constant spectral acceleration range

T_{d1} fundamental period of the structure with energy dissipation devices when the primary seismic elements of the main structural system reach the design displacement d_{Ds} at each storey *s*

T_{eff} effective period

T_{eff1} effective period in the first mode

T_{fm} *m*-th mode vibration period of the main structural system in elastic conditions, exclusive of energy dissipation system

T_m *m*-th mode vibration period of the structure with energy dissipation system in elastic conditions

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T_n period of the n^{th} mode of the structure in the considered horizontal direction

$T_{p,i}$ natural period of the i^{th} mode of the structure; $T_{p,1}$ corresponds to T_1 (in s)

T_{p1} fundamental period of the primary seismic elements of the main structural system in elastic conditions, exclusive of energy dissipation system

T_{s1} period that gives the maximum input energy between T_{d1} and $1{,}4T_{d1}$

T_v fundamental vibration period in the vertical direction

V shear force

$V_{Rd,b}$ design shear resistance of a steel beam cross-section

V_c shear force applied to a column

$V_{Rd,c}$ design shear resistance of a steel column cross-section

V'_{Edw} design shear force of the wall in the seismic design situation as obtained from the analysis

$V'_{Edw,1}$ design shear force of the wall due to the mode with the largest participating mass, in the direction of analysis of the structure, as obtained from the modal response spectrum analysis

V_{dys} *s*-th storey yield shear of the energy dissipation system

V_{Ed} design shear force from the analysis in the seismic design situation

V_{Edcb} shear in the coupling beam calculated from the analysis

$V_{Ed,E,LLRS,i}$ design global shear of the i^{th} storey due to the seismic action

$V_{Ed,E,i,j}$ design global shear of the j^{th} segmented timber shear wall at the i^{th} storey, due to seismic action

$V_{Ed,M}$ Design shear coherent with plastic hinges present at both ends of a member

V_{Edw} diagram of magnified shear force applied to the wall

$V_{Edw,base}$ design shear at the base of the wall in the seismic design situation (envelope)

$V_{Edw,env}$ design shear of the wall in the seismic design situation (envelope)

$V_{Edw,top}$ design shear at the top of the wall in the seismic design situation (envelope)

V_{fD1} maximum base shear that can be endured by the primary seismic elements of the main structural system within the linear elastic range

V_{fDs} maximum shear that can be endured by the primary seismic elements of the main structural system at the *s*-th storey within the linear elastic range

V_{fys} *s*-th storey yield shear corresponding to the formation of a full plastic mechanism in the primary seismic elements of the main structural system

V_i shear force in pier *i*

$V_{i,d}$ design shear force at beam or column end *i*

$V_{ap,Rd}$ in-plane design resistance in shear of an infill panel

$V_{Ed,E,i}$ total design shear load acting on the shear-wall at the i^{th} storey due to seismic action $V_{p,Rd}$

$V_{p,link}$ design value of shear resistance of a link in eccentrically braced frames

$V_{pl,Rd}$ design value of shear resistance of a steel member

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V_{Rd} design shear resistance

$V_{Rd,a,i,j}$ design lateral strength related to shear connections of the j^{th} segmented timber shear wall at the i^{th} storey

$V_{Rd,c}$ design shear resistance of concrete without shear reinforcement

$V_{Rd,LLRS,i}$ design lateral strength of the primary structure at the i^{th} storey

$V_{Rd,int}$ resistance of the integrity reinforcement in the flat slab

$V_{R,peak}$ peak shear strength of an unreinforced masonry spandrel

$V_{R,lim}$ absolute maximum value of the peak shear strength $V_{R,peak}$

$V_{Rd,MRF}$ shear resistance of the moment resisting frames at the building base

$V_{Rd,sh,i,j}$ design lateral strength related to connections between sheathing material and timber frame of the j^{th} shear wall at the i^{th} storey

$V_{Rd,total}$ shear resistance of the whole structure at the building base

$V_{Rd,walls}$ shear resistance of the walls at the building base

$V_{s,SD}$ total storey shear at SD limit state calculated from the analysis for the displacement-based approach

V_{storey} storey shear action effect

V_{tot} total shear at a given storey in the seismic design situation

$V_{wp,Ed}$ design shear force in the web panel due to the action effects

$V_{wp,Rd}$ shear resistance of the web panel

$V_{wb,Rd}$ shear buckling resistance of a web panel

$W_{pl,y}$ plastic section modulus

3.2.1.2 Lower case Latin symbols

an index, corresponding to the ancillary element

a slab overhang

a_{CLT} smaller size (width) of a hole in a CLT floor panel

a_{pj} clear distance between the inner and outer gusset plates in INERD pin connection

b cross-section width of a beam or column

b_0 length of the control perimeter for punching shear resistance verification

b_{0C} smallest dimension of the concrete core (to the centreline of the hoops)

b_b cross-section width of a beam

b_c cross-section width of a column

b_{CLT} length of each single panel making a shear wall; longer size (length) of a hole in a CLT floor panel

b_{ef} effective width

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$b_{eff,Rd}$ partial effective width of the slab for the calculation of plastic resistance of composite sections

b_f wall flange thickness or steel beam flange width

b_{int} control perimeter activated by the integrity reinforcement in a flat slab

$b_{j,ef}$ effective joint width

b_{max} largest cross-sectional dimension of a column

b_{pin} width of the pin section in INERD pin connection

b_{sp} correction coefficient in the calculation of masonry spandrel strength

b_w wall thickness

b_{w0} wall web thickness

c outstand of a steel flange, also named the side width of a steel member

c_E correction factor in pushover analysis which primarily accounts for higher-mode effects in elevation

$c_{E,i}$ correction factor c_E for the i-*th* storey of the structural member

c_f factor accounting for the masonry typology

c_{hd} distance from the anchoring connections against overturning to the nearest vertical edge of a CLT panel

c_P correction factor in pushover analysis which primarily accounts for the torsional effects

c_{pj} distance between the outer gusset plates in INERD pin connection

$c_{P,j}$ correction factor c_P for the j-*th* location of a structural member in plan

c_{v1} coefficient for the determination of the shear resistance at the interfaces

d lateral elastic displacement of the top of the building (in m) due to the gravity loads applied in the corresponding horizontal direction

d_b depth of a beam section

d_{bolt} nominal diametre of a bolt

d^* displacement of the equivalent SDOF system

d_{bL} longitudinal bar diameter

$d_{bL,min}$ minimum longitudinal bar diameter

d_{bw} hoops diameter or straight links diameter

d_c depth of a section of a column or of a composite beam

d_{dg} size parameter describing the crack and failure zone roughness, taking account of concrete type and its aggregate properties

d_{Ds} maximum interstorey drift that can be sustained by the primary seismic elements of the main structural system at the *s*-th storey within the linear elastic range

d_{dys} *s*-th storey drift at yielding of the energy dissipation devices of this storey

$d_{e,jt}$ value of the displacement at the *j*-th location in plan, obtained from the linear elastic analysis associated with the design seismic action

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d_{et} value of the control displacement obtained from the linear elastic analysis associated with the design seismic action

d_{fa} fastener diameter

$d_{f,y}$ yielding displacement of a structure with velocity-dependent energy dissipation devices, relative to the ground, at roof level

d_{fys} *s*-th storey drift corresponding to the formation of a full plastic mechanism in the primary seismic elements of the main structural system

d_{i1} maximum horizontal displacement at the *i*-th floor relative to the ground in the first mode

d_n total lateral displacement of the shear-wall at the top of the n^{th} storey

d_r design interstorey drift, evaluated as the difference of the average lateral displacements d_s at the top and bottom of the storey under consideration and calculated in accordance with 6.4.1

$d_{r,DL}$ design interstorey drift at DL limit state

$d_{r,SD}$ design interstorey drift at SD limit state

$d_{ret,i}$ interstorey drift at the centre of mass of the *i*-th storey, obtained from linear elastic analysis associated with the design seismic action

$d_{roof,y}$ yielding displacement of a structure with displacement-dependent energy dissipation devices, relative to the ground, at roof level

d_{roof1} maximum displacement relative to the ground at roof level for inelastic behaviour of the structure with energy dissipation system, including the elastic part, in the first mode

d_{roof1e} maximum displacement relative to the ground at roof level in the first mode for a structure with energy dissipation in the elastic range

d_{rs1} maximum interstorey drift at the *s*-th storey

$d_{rt,i}$ interstorey drifts at the centre of mass of the *i*-th storey which corresponds to the target displacement associated with the considered limit state

$d_{t,j}$ displacement at the *j*-th location in plan, if the control displacement is equal to the target displacement d_t

d_{sp} spandrel depth

$d_{SD,\theta}$ displacement of a control node at which a primary pier first reaches its resistance

d_v shear-resisting effective depth of the slab or drop panel

d_y^* displacement of a control node at which a primary pier first reaches its resistance

e length of a link in an eccentric bracing

e_{bp} base plate eccentricity

e_{crit} critical eccentricity of an exposed column base

e_{ox} distance between the centre of stiffness and the centre of mass, measured along the *x* direction

f_{ap} design lateral load per unit area in a panel or an infill

f_b normalized compressive strength of masonry units

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f_{bv} normalized compressive strength of masonry units normal to bed face

f_{bh} normalized compressive strength of masonry units parallel to bed face

$f_{c,eff,CLT,i,j}$ effective compressive design strength of the j^{th} CLT shear wall at the i^{th} storey

f'_{cd} increased joint concrete compressive strength due to confinement by joint reinforcement

f_{cd} design compressive strength of a concrete cylinder

f_{ck} characteristic compressive strength of a concrete cylinder

f_{ctm} mean tensile strength of concrete

f_k characteristic compressive strength of masonry

f_{kt} characteristic shear strength associated to diagonal cracking of a masonry spandrel

f_u ultimate strength of steel

$f_{u,k}$ characteristic ultimate tensile strength of the metal fastener

f_y nominal yield strength of constructional steel

f_{yd} design yield strength of steel reinforcement

f_{yk} characteristic yield strength of steel reinforcement

$f_{y,mean}$ mean yield strength of constructional steel

$f_{y,max}$ specified upper value of yield strength of constructional steel

$f_{y,dl}$ design yield stress of the longitudinal rebars in a composite section

$f_{y,dw}$ design yield stress of the confining hoops in a composite section

$f_{y,df}$ design yield stress of the flange of the steel cross-section in a composite section

f_{ywd} design yield strength of the transverse reinforcement

$f_{ywd,j}$ design yield strength of the transverse reinforcement in the joint

f_{vk0} characteristic initial shear strength of masonry under zero compressive stress

f_0 conventional elastic strength of aluminium

h cross-section depth of a beam or column

h_0 largest dimension of the concrete core to the centreline of the hoops

h_b cross-section depth of a beam

h_c cross-section depth of a column

h_{cr} height of the wall critical region

h_{ef} effective height of a masonry as defined in EN 1996-1-1

$h_{j,ef}$ effective joint depth

h_{min} free height of the shortest pier

h_{op} clear height of an opening adjacent to a masonry pier

h_{ap} length of an infill wall

h_{pier} clear height of a pier as in EN 1996-1-1:2022, 7.5.1.3(9)

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h_{pin}	height of the pin section used in INERD pin connection
h_{RB}	depth of reinforced concrete ring beams
h_s	storey height (also named interstorey height)
$h_{s,cl}$	clear storey height
h_{sp}	length of a spandrel
h_{ss}	distance between the upper flanges of steel profiles or the tops of floor slabs at each of the levels above and below a splice
h_w	wall height
i_z	radius of gyration referring to the weak axis of a steel cross-section
k	ratio of characteristic tensile strength to yield strength of reinforcement
k_{16}	ratio between the 16th and the 5th percentiles of the resistance distribution
k_{br}	vertical stiffness of a steel bracing
k_D	factor for the anchorage verification of beam reinforcement in joints, reflecting the ductility class
k_{deg}	strength reduction factor due to degradation under cyclic loading
k_{IR}	amplification factor that applies on seismic action for masonry infills in case of structural irregularity in elevation
k_{mean}	ratio between the mean and the characteristic strength in static conditions
k_{mod}	modification factor for duration of load and moisture content for the strength of timber
k_{stif}	minimum value over all stories of the ratio between the horizontal stiffness of the CLT walls and the total horizontal stiffness of the hybrid primary structure in each main direction at the i^{th} storey
k_t	factor of reduction of the design shear resistance of connectors related to the shape of profiled steel sheeting supporting a concrete slab
k_w	factor accounting for the prevailing failure mode in large walls structures
k_y	ratio between the yield and maximum strength of dissipative connections
l_1, l_2	slab spans
l_0	distance between the centrelines of two walls
l_b	anchorage length
l_c	length of the boundary elements in the wall
l_{cl}	beam or column clear length
l_{cs}	length of a column over which the diagonal strut force of an infill is applied
l_{CLT}	width of a CLT floor panel
l_{cr}	beam or column critical region length
l_f	wall flange width
$l_{infill,left,x,i}$	length of an infill parallel to axis x situated at left of the axis

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$l_{infill,right,x,i}$ length of an infill parallel to axis x situated at right of the axis

l_{mp} length of a masonry pier or wall

l_{op} length of an opening

l_{ap} height of an infill wall or panel

l_s length of an infill panel diagonal

$l_{s,i}$ radius of gyration of floor i

l_w wall length

m total mass of the building above the foundation or above the top of a rigid basement

m^* mass of an equivalent SDOF system

m_{an} mass of an ancillary element

$m_{eff,j}$ effective modal mass corresponding to the j-th horizontal direction of loading

m_{eff1} first mode effective mass of the structure with energy dissipation system in the elastic range

$m_{f,eff1}$ first mode effective mass of the main structural system in the elastic conditions, exclusive of the energy dissipation system

m_{lp} number of CLT panels in a segmented wall

m_i mass of storey or floor or level i

m_s factor for the determination of the shear magnification factor in the wall

$\overline{m}_s$ mass above the s-th storey normalized by the total mass of the building

n_{vj} number of fasteners used in the vertical joint

n value representing the extent to which the required amount of energy absorption of the building is distributed to each storey according to the stiffness and strength of each storey

n_0 modular ratio equal to E_a/E_{cm}

n_e number of fasteners in the outer circle of a moment transmitting joint

p index corresponding to the primary structure

p_A minimum ratio of the area of masonry walls in one direction

$p_{A,min}$ absolute minimum ratio of the area of masonry walls in one direction

$p_{A,ref}$ reference ratio of the area of masonry walls in one direction

p_s deviation of the s-th storey seismic shear from an optimum value that would provide an even distribution of damage among storeys

P_{tot} total gravity load at and above the storey considered in the seismic design situation

p_{ts} coefficient that accounts for the increase in the required energy absorption capacity in a storey s resulting from torsional effects

q behaviour factor

q_a behaviour factor of an ancillary element

q_D behaviour factor component accounting for the deformation capacity and energy dissipation capacity

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$q_{D,CLT}$	default value for the reference Ductility Class of the component q_D of the behaviour factor q for a CLT primary structure
$q_{D,Hyb}$	value of the component q_D of the behaviour factor q for a hybrid framed wall-CLT primary structure
$q_{D,FW}$	default value for the reference Ductility Class of the component q_D of the behaviour factor q for the framed wall primary structure
q_{oop}	behaviour factor for out-of-plane bending of masonry walls
q_R	behaviour factor component accounting for overstrength due to the redistribution of seismic action effects in redundant structures
q_S	behaviour factor component accounting for overstrength due to all other sources
r_i	minimum torsional radius of the *i*-th floor
r_x	square root of the ratio of the torsional stiffness to the lateral stiffness in the *y* direction ("torsional radius")
$r_{x,I}$, $r_{y,I}$	x component of the torsional radius (respectively y component)
s	spacing of transverse shear or punching shear reinforcement
s_h	distance between the centre of a plastic hinge and the column axis
s_i	displacement of storey *i* under horizontal unitary acceleration
s_{int}	width of the integrity bars group in a flat slab
s_{lap}	spacing of the transverse reinforcement in a lap zone
s_s	ratio of the required amount of energy absorption on storey *s* to the required amount of energy absorption on the first storey
t	thickness
$t_{b,f}$	thickness of a beam flange
$t_{c,f}$	thickness of a column flange
t_{ef}	effective thickness of a masonry wall as defined in EN 1996-1-1
$t_{eff,CLT,i,j}$	effective compressive thickness of the j^{th} CLT shear-wall at the i^{th} storey
t_f	steel flange thickness
t_{gp}	thickness of a gusset plate
t_p	thickness of an infill panel
t_w	anchorage depth of a smooth nail
t_{RB}	width of reinforced concrete ring beams
t_{sp}	masonry spandrel thickness
t_w	masonry wall thickness / thickness of the web of a steel profile
t_{wsp}	thickness of a supplementary web plate
u_{ult}	interstorey displacement in direction *x* (respectively *y*) at the pier where the shear force is incremented, corresponding to the attainment of the ultimate rotation in any pier of the storey along direction *x* (respectively *y*)

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w_{gp} effective width of a gusset plate

w_s width of the compression strut in an infill panel

w_w wall length in light weight steel structures

$w_{w,eff}$ effective wall length in light weight steel structures

x distance of the element under consideration from the centre of mass of the building in plan, measured perpendicularly to the direction of the seismic action considered

x_c length of the compression zone

z distance along the wall height

z_c plastic neutral axis depth of a composite section

z_i, z_j heights of the masses m_i and m_j above the level of application of the seismic action (foundation or top of a rigid basement)

3.2.1.3 Upper case Greek symbols

Γ_1 first mode participation factor of a structure with energy dissipation system in the elastic range

Γ_i modal participation factor for the i^{th} mode

Γ_{f1} first mode participation factor of the main structural system in elastic conditions, exclusive of the energy dissipation system

Γ_m m-th mode participation factor of a structure with energy dissipation system in the elastic range

Δd_{j1} relative displacement between each end of the energy dissipation device measured along its axis in the first mode

Δv_{j1} relative velocity between each end of the energy dissipation device j measured along its axis in the first mode

$\Delta F_{1\text{-}3}$ impairment of strength, namely the load reduction $\Delta F_{1\text{-}3}=F_1-F_3$ when attaining a certain deformation δ from the first to the third cycle of the same amplitude in a cyclic test performed in accordance with EN 12512

$\Delta\varphi$ fraction of the critical section with the maximum punching shear stress

$\Delta V_{max,I}$ maximum value of the reduction or increase ΔV of the absolute value of the shear force in a pier resulting from a redistribution of forces

ΔV_{Rw} difference between the shear resistance of two storeys, related to the quantity of infills in each storey

ΔM_{Ed} variation of the moment in a column adjacent over one storey height

$\sum M_{Rb}$ sum of the design values of the moments of resistance of the beams framing the joint

$\sum M_{Rc}$ sum of the design values of the moments of resistance of the columns framing the joint

ΣV_{Ed} sum of the calculated seismic shears in all vertical primary seismic members of a storey

ϕ_{ij} value of the i^{th} mode shape at floor j

ϕ_1 vector of first mode of vibration normalized so that its component at roof level is equal to 1

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ϕ_{i1} component of the first mode vector (normalized so that at roof level the value is equal to 1) at the *i*-th floor of the structure with energy dissipation system in the elastic range

ϕ_m vector of *m*-th mode of vibration normalized so that its component at roof level is equal to 1

Ω seismic action magnification factor

$\Omega_{d,i}$ ratio of the design resistance R_{di} to the design action effect E_{di} in member *i* or in storey *i*

Ω_d particular value of $\Omega_{d,i}$ among all $\Omega_{d,I}$ and applied in the design of specific parts of a structure

$\Omega_{d,M}$ design overstrength of a pin connection yielding in bending

$\Omega_{d,V}$ design overstrength of a pin connection yielding in shear

3.2.1.4 Lower case Greek symbols

α angle of a brace with the horizontal

α_0 prevailing aspect ratio, i.e. prevailing height-to-length ratio of walls

α_1 value by which the horizontal seismic design action is multiplied, in order to first reach the plastic resistance in any member of the structure, while all other design actions remain constant

α_{jo} joint aspect ratio

α_{SD} ratio between resistance at the SD limit state and the ultimate resistance in terms of generalized displacements

α_u value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural (plastic) instability, while all other design actions remain constant

α_{ult} angle between the integrity bars and the flat slab plan at failure

α_{ve} velocity exponent of a velocity-dependent energy dissipation device

γ_a performance factor of ancillary element

γ_c partial safety factor for concrete

γ_{CT} compression strength adjustment factor in buckling restrained braces

γ_d overstrength factor which is used for multiplication of the seismic action effects in the diaphragm obtained from the analysis

$\gamma_{DL,CC}$ performance factor for DL limit state and a Consequence Class CC

γ_M partial safety factor for material properties in the seismic design situation

$\gamma_{M,SD}$ partial factors on resistance of masonry

γ_{Rd} overstrength factor used for reinforced concrete and timber design

$\gamma_{Rd,NC}$ partial factors on resistance at NC limit state

$\gamma_{Rd,SD}$ partial factors on resistance at SD limit state

γ_s partial safety factors for reinforcing steel

γ_{SB} partial safety factor in the calculation of the required area of masonry walls

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$\gamma_{s,SD}$	partial factors on resistance at SD limit state for reinforcing steel
γ_x	amplification factor for energy dissipation devices
δ_{lag}	phase lag angle
δ	generalized deformation (displacement, rotation)
δ_{Fmax}	deformation at maximum load of a dissipative timber connection
δ_{NC}	deformation at NC of a dissipative connection under seismic loading
δ_{SD}	deformation at SD of a dissipative connection under seismic loading
δ_u	ultimate deformation of a dissipative connection under seismic loading
δ_y	yield deformation of a dissipative connection under seismic loading
ε	shear magnification factor for walls
ε_a	total strain of the steel profile in a composite section
ε_{cu}	unconfined concrete ultimate strain
ε_{cu2}	ultimate compressive strain of concrete considering confinement
η_{loss}	loss factor of a viscoelastic energy dissipation device
θ	interstorey drift sensitivity coefficient
θ_e	drift ratio or chord rotation of a masonry member
θ_i	angle between the member axis at end section *i* and the chord line that joins the centroids of the two end sections
θ_j	angle of inclination of energy dissipation device *j* to the horizontal
θ_p	plastic rotation
θ_{SD}	chord rotation capacity of a pier
θ_{tot}	rotation capacity of strong and weak partial strength joints for moment resisting frames
θ_{u2}	deformation capacity in terms of member drift at 20 % drop in strength
θ_y	chord rotation at yield
θ^*	largest chord rotation in the shortest pier
λ	correction factor used in the lateral force method of analysis
λ_{ns}	coefficient accounting for sensitivity of ancillary elements to interstorey drift at DL
λ_s	coefficient reflecting the limitation of drift at SD
$\bar{\lambda}$	non-dimensional slenderness of columns
μ	ductility of a dissipative connection under seismic loading
μ_f	displacement ductility ratio of a structure with velocity-dependent energy dissipation devices
ν	strength reduction factor for cracked concrete
ν_d	normalized axial load

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$\nu_{d,c}$	normalized axial load in composite walls
$\nu_{E,k}$	axial load ratio for the design seismic action, calculated as $\nu_{E,k} = N_{Ed}/(A_p f_k)$
$\nu_{G,k}$	axial load ratio for the non-seismic actions in the seismic design situation, calculated as $\nu_{G,k} = N_{Ed,G}/(A_p f_k)$
$\nu_{G,k,av}$	average axial load ratio of a set of piers
ξ_a	damping value of the ancillary element in % of critical damping
ξ_{eff}	effective damping
ξ_{eff1}	effective damping in the first mode
ξ_{effm}	effective damping in the m-th mode
ξ_H	component of effective damping of the structure due to post-yield hysteretic behaviour of the primary seismic elements of the main structural system
ξ_I	inherent damping ratio
$\xi_{p,i}$	damping value of the primary structure for buildings in % of critical damping
ξ_{V1}	component of effective damping of the first mode of the structure due to viscous dissipation of energy by the damping system at or just below the yield displacement of the primary seismic elements of the main structure
ρ'_l	longitudinal reinforcement ratio in the compression zone
ρ_l	longitudinal reinforcement ratio
$\rho_{l,max}$	maximum longitudinal reinforcement ratio
$\rho_{l,min}$	minimum longitudinal reinforcement ratio
ρ_n	factor related to the effective height as in EN 1996-1-1:2022, 7.5.1.3(10)
$\rho_{v,min}$	minimum vertical reinforcement ratio in the vertical joint between precast panels relatively to the area of grout
ρ_w	shear reinforcement ratio
$\rho_{w,min}$	minimum shear reinforcement ratio
ρ_{wj}	transverse reinforcement ratio of the joint
σ_{cm}	mean value of the concrete stress in the compression zone of the section of the wall
σ_{Ed}	stress level
τ_{Ed}	design punching shear stress for the seismic design situation
$\tau_{Ed,grav}$	punching shear stress for the gravity loads design situation
τ_{Edj}	joint shear stress
$\tau_{Rd,c}$	design resistance of concrete to punching shear stress for members without shear reinforcement
τ_{Rdj}	joint shear resistance
$\tau_{Rd,max}$	maximum punching shear stress resistance of planar members with shear reinforcement
φ_{imp}	strength impairment factor of a dissipative timber connection

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ψ_{max} maximum slab rotation relatively to the slab support connection

ω_{pb} multiplicative factor on design value $N_{pl,Rd}$ of yield resistance in tension of compression brace, for the estimation of the residual post-buckling resistance of the brace.

ω_{rm} ratio between the expected average yield strength of steel products of a certain steel grade and the nominal yield strength of that steel grade; for aluminium, ratio of the expected (i.e. average) value of the conventional elastic limit f_o to its design value from EN 1999-1-1

ω_{sh} factor taking into account the strain-hardening effect of steel or aluminium

ω_{wd} mechanical volumetric ratio of confining hoops

3.2.2 Abbreviations

AAC Aerated autoclaved concrete

CLT Cross-laminated timber

GF Gypsum fibre board

GLVL Wood based composite which is produced from several LVL panels by face gluing (glued laminated veneer lumber)

KLD Knowledge level on details

LVL Laminated veneer lumber

LVL-C Symmetrically arranged structural laminated veneer lumber comprising at least two crossband veneers

MRF Moment resisting frame

NC Near Collapse limit state

OP Fully operational

OSB Oriented strand board

PFA Peak floor acceleration

SD Significant Damage limit state

SDOF Single-degree-of-freedom

SWP Solid wood panels

1D One-dimensional

2D Two-dimensional

3D Three-dimensional

3.3 S.I. Units

S.I. Units in accordance with ISO 80000 shall be used.

For calculations, the following units should be used when applicable:

— mass: kg, t

— mass density: kg/m^3, t/m^3

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- unit mass: kg/m^3, t/m^3
- forces and loads: kN, kN/m, kN/m^2
- weight density: kN/m^3
- stresses and strengths: Pa (= N/m^2), kPa (= kN/m^2), MPa (= MN/m^2)
- moments (bending, etc.): kNm
- acceleration: m/s^2
- length: m, mm
- modulus of elasticity: MPa, GPa
- angle: degree, mrad

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4 Basis of design

4.1 Performance requirements

(1) The performance requirements shall refer to the state of damage in the structure, herein described through the Limit States (LS) defined in prEN 1998-1-1:2022, 4.3(1).

(2) Consequence class CC3 should be divided into CC3-a and CC3-b according to prEN 1998-1-1:2022, 4.2(3).

NOTE 1 The definitions of buildings included in CC3-a and CC3-b are given in Table 4.1 (NDP), unless the relevant authorities or the National Annex give different definitions for use in a country.

Table 4.1 (NDP) — Definitions of consequence classes CC3-a and CC3-b for buildings

CC3-a	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse (e.g. schools, assembly halls, cultural institutions)
CC3-b	Buildings of installations of vital importance for civil protection (e.g. hospitals, fire stations, and their equipment), and buildings which should remain operational at all times (e.g. communication centres, data centres, and their equipment)

NOTE 2 Buildings that house very dangerous installations or materials are generally classified as CC4, which is not fully covered by this standard.

(3) For the application of prEN 1998-1-1:2022, 4.1(4), δ values shall be determined.

NOTE The values of δ applicable to buildings are those given in Table 4.2 (NDP), unless the relevant authorities or the National Annex give different values for use in a country.

Table 4.2 (NDP) — δ values for buildings

	Consequence class			
	CC1	CC2	CC3-a	CC3-b
δ	0,6	1,0	1,25	1,6

4.2 Seismic actions

(1) prEN 1998-1-1:2022, 4.3(3) and (4), should be applied.

NOTE 1 The values of return period $T_{LS,CC}$, according to prEN 1998-1-1:2022, 4.3(3), are those given in Table 4.3 (NDP). The values in Table 4.3 (NDP) are computed with the values of $\beta_{t,LS,CC}$ suggested in prEN 1998-1-1:2022, F.3. If a relevant authority or the National Annex give different values of $\beta_{t,LS,CC}$ for use in a country, values of $T_{LS,CC}$ should be updated according to the note in prEN 1998-1-1:2022, 4.3(3).

NOTE 2 When performance factors are used, according to prEN 1998-1-1:2022, 4.3(4), the values of $\gamma_{LS,CC}$ are those given in Table 4.4 (NDP). The values in Table 4.4 (NDP) are consistent with those in Table 4.3(NDP). If a relevant authority or the National Annex give different values of $\beta_{t,LS,CC}$ for use in a country, values of $\gamma_{LS,CC}$ can be updated according to the note in prEN 1998-1-1:2022, 4.3(5).

NOTE 3 When the OP limit state is required for CC3 buildings and their equipment, and unless the relevant authorities or the National Annex give different values for use in a country, or, in the absence of such requirements, these values are agreed for a specific project by the relevant parties, then:

— the return periods and performance factors for the structure are the same as under the DL limit state,

— the return period and performance factor for equipment are 475 years and 1, respectively.

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Table 4.3 (NDP) — Return period $T_{LS,CC}$ values, in years, for buildings

Limit state	Consequence class			
	CC1	CC2	CC3-a	CC3-b
NC	500	1350	2500	5000
SD	275	475	700	1000
DL	100	115	125	140

Table 4.4 (NDP) — Performance factor $\gamma_{LS,CC}$ values for buildings

Limit state	Consequence class			
	CC1	CC2	CC3-a	CC3-b
NC	1,00	1,40	1,70	2,20
SD	0,85	1,00	1,15	1,30
DL	0,60	0,60	0,65	0,65

(2) If $S_{\alpha v}$ is greater than 6 m/s², the vertical component of the seismic action, as defined in prEN 1998-1-1:2022, 5.2.2.3, should be considered for the verification of the following primary or secondary seismic members, and their directly associated supporting structural members, in a) to e):

a) horizontal or nearly horizontal structural members spanning longer than 20 m;

b) horizontal or nearly horizontal cantilever components longer than 5 m;

c) horizontal or nearly horizontal pre-stressed components;

d) beams supporting columns;

e) all structural members of the primary structure of transfer zones (see 6.2.11).

(3) The value of β introduced in prEN 1998-1-1:2022, 6.4.1(6), should be taken equal to 0,08 for buildings.

4.3 Compliance criteria

(1) Unless 14.8 is used for "simple masonry buildings", the seismic performance of buildings should be verified according to Clause 6.

NOTE Specific rules for "simple masonry buildings" are given in 14.8, where they are defined. By conforming to those rules, such "simple masonry buildings" are deemed to satisfy the performance requirements of prEN 1998-1-2 without analytical safety verifications.

4.4 Characteristics of earthquake resistant buildings

4.4.1 Conceptual design

(1) A building structure shall be able to resist horizontal actions in any direction.

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(2) Seismic performance of a building should be considered in the early stages of conceptual design, achieving a structural system which, with acceptable costs, satisfies the performance requirements specified in prEN 1998-1-1:2022, 4.1.

NOTE Guidance for good practice is given in informative Annex A.

(3) The seismic action resisting system of a building may belong to one of the structural types in 10.4.1 for reinforced concrete buildings, in 11.4.1 for steel buildings, in 12.4.1 for composite steel-concrete buildings, in 13.4.1 for timber buildings, in 14.2.2 for masonry buildings, in 7.4 for masonry infilled frames and in 15.4.1 for aluminium buildings.

(4) Seismic action resisting systems may be different, and made of different materials, in two main directions of a building.

(5) Hybrid structures, combining in the same plane two or more of the types of structural systems as given in (3), may be used.

(6) Seismic action resisting systems different from those in (3) may be used, if they satisfy 10.4.1(6) for reinforced concrete buildings, 11.4.1(3) for steel buildings, 12.4.1(6) for composite buildings, 13.4.1(4) for timber buildings and 15.4.1(3) for aluminium buildings.

4.4.2 Primary and secondary seismic members

(1) A certain number of structural members (e.g. beams and/or columns) may be designated as secondary seismic members, not forming part of the seismic action resisting system of the building. The strength and stiffness of these members against seismic actions should be neglected in the models for seismic analysis.

(2) The secondary seismic members and their connections should be designed and detailed to maintain support of gravity loads when subjected to the displacements caused by the most unfavourable seismic design situation. Due allowance of second-order effects (P-Δ effects), in both the structure and these members, should be made in the design of these members.

(3) Secondary columns which yield under the displacements caused by the seismic design situation in (2) should satisfy the requirements for local ductility of columns in DC2 given in 10.5.4.2 for reinforced concrete, in 11.8.3 for steel, in 12.8.3 for composite and in 15.8.3 for aluminium; connections in timber structures should satisfy 13.2.1(7) and (8).

(4) The designation of some structural members as secondary seismic members should not change the classification of the structure from non-regular to regular as described in 4.4.3 and 4.4.4.

(5) All structural members not designated as being secondary seismic members should be taken as primary. They should be modelled in the structural analysis in accordance with 5.1 and designed and detailed for earthquake resistance in accordance with 10 to 15.

NOTE The primary structure is composed of the set of primary seismic members over the building. A primary bracing is a set of primary members installed in a vertical plane at a given level of the building.

(6) The contribution of lateral stiffness of secondary members to the total stiffness should not affect significantly the dynamic behaviour of the structure. To fulfil this requirement, either a) or b) should be satisfied:

a) The total contribution to lateral stiffness of all secondary seismic members does not exceed 15 % of that of all primary seismic members;

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b) The total contribution to lateral stiffness of all secondary seismic members does not exceed 30 % of that of all primary seismic members and two analysis are performed: one with only the primary members in the model, the other one including all primary and secondary members in the model; the most unfavourable seismic action effects resulting from both analyses should be considered in the verification of the primary seismic members and in the capacity design of the members and connections adjacent to those primary seismic members.

(7) The primary structure should be assigned a ductility class according to prEN 1998-1-1:2022, 4.5.2(3). The ductility class should be unique for the building.

4.4.3 Torsionally flexible buildings

(1) A building should be considered torsionally flexible if it satisfies a) or b):

a) in each main horizontal direction, the greatest effective modal mass is not that of the first or second mode, possible local modes and equipment modes not being considered in this classification;

NOTE This condition corresponds to the fact that the first mode (in at least one horizontal direction) is substantially influenced by torsion.

b) the period of the first predominant torsional mode of vibration is greater than the periods of the predominant translational modes in the two main horizontal directions, possible local modes and equipment modes not being considered in this classification.

(2) Buildings verifying Formula (4.1) at every storey i, with the possible exception of the highest storey, may be considered as not being torsionally flexible.

$$\frac{r_{\mathrm{i}}}{l_{\mathrm{s,i}}} \geq 1 \tag{4.1}$$

where

r_{i} is the minimum torsional radius of the i-th floor, equal to the square root of the ratio of the torsional stiffness, with respect to the centre of lateral stiffness C_{i} (Figure 4.1), to the largest of the floor lateral stiffnesses: $r_i = \min\left(r_{x,i}, r_{y,i}\right)$;

$l_{\mathrm{s,i}}$ is the radius of gyration of floor i: ${l_{\mathrm{s,i}}}^2 = \frac{I_{\mathrm{x,i}}+I_{\mathrm{y,i}}}{m_{\mathrm{i}}}$; in the case of approximately rectangular floor with a well distributed mass, ${l_{\mathrm{s,i}}}^2$ may be taken as ${l_{\mathrm{s,i}}}^2 = \frac{{L_{\mathrm{i}}}^2+{B_{\mathrm{i}}}^2}{12}$;

$I_{\mathrm{x,i}}$ and $I_{\mathrm{y,i}}$ are the mass moments of inertia of the floor around axis x and y respectively, going through the floor centre of mass G_{i};

m_{i} is the mass of floor i;

L_{i} and B_{i} are the dimensions of the rectangular floor.

NOTE Annex B gives procedures to calculate $r_{x,i}$ and $r_{y,i}$.

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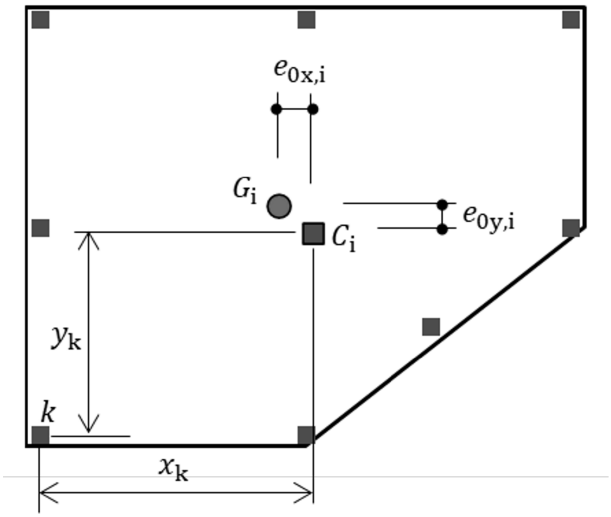


Figure 4.1 — Relative position of each primary resisting member k with respect to the centre of lateral stiffness C_i and relative position of the latter with respect to the centre of mass G_i

4.4.4 Structural regularity

4.4.4.1 General

(1) Although not mandatory, achieving building regularity should be regarded as a good practice for the design of earthquake resistant buildings.

NOTE Guidance for in-plan regularity is given in informative Annex A, while criteria for regularity in elevation are given in 4.4.4.2.

4.4.4.2 Regularity in elevation

(1) For a building to be categorized as being regular in elevation, it should satisfy all the conditions listed in a) to c), considering that criteria in b) and c) apply to the structure above the top of the foundation or the top of a rigid basement when it exists:

a) all primary members, such as cores, structural walls or frames and the diaphragms which connect them, provide a continuous resisting system without interruption from the top of the foundation or the top of a rigid basement to the top of the building or, if setbacks at different heights are present, to the top of the corresponding zone of the building.

b) both the lateral stiffness and the mass of the individual storeys remain constant or decrease gradually by no more than 20 % relative to the storey below, without abrupt changes, from the base to at least one storey below the top storey.

c) the ratio of the actual storey resistance to the resistance required by the analysis does not vary by more than 30 % between adjacent storeys.

NOTE Within this context, the special aspects of masonry infilled frames are treated in Clause 7.

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5 Modelling and structural analysis

5.1 Modelling

5.1.1 General

(1) The model of the building should comply with prEN 1998-1-1:2022, 6.2.

(2) Seismic analysis may be performed using two planar models, one for each main horizontal direction, if the height of the single or multi-storey building does not exceed, respectively, 15 m or 10 m, and if the floors may be considered as rigid in their planes according to 5.1.3(6). In that case, the structural (natural) mass eccentricity, multiplied by 1,1, should be taken into account in both directions for the calculation of the seismic action effects.

5.1.2 Masses

(1) The combination coefficients ψ_{Ei} in prEN 1998-1-1:2022, 6.2.1(3), for the masses associated to variable actions should be calculated with Formula (5.1) where ϕ values should not be smaller than those given in Table 5.1.

$$\psi_{\mathrm{Ei}} = \phi\, \psi_{2\mathrm{i}} \tag{5.1}$$

Table 5.1 — Minimum values of ϕ for calculating ψ_{Ei}

Type of variable action	ϕ
Categories* A and C	0,5
Other categories	1,0
* Categories as defined in prEN 1991-1-1:2023, Table 8.1.	

(2) When the floor and roof diaphragms or bracings of the building may be considered as being rigid in their planes according to 5.1.3(6), the masses and the mass moments of inertia of each of those planes may be lumped at the centre of gravity.

5.1.3 Stiffness

(1) The stiffness of secondary seismic members against seismic actions should be neglected if the condition in 4.4.2(6) a) is satisfied; it should be taken into account according to 4.4.2(6) b) if the condition in 4.4.2(6) a) is not satisfied.

(2) In concrete buildings, in composite steel-concrete buildings, in masonry buildings and in hybrid buildings as defined in 4.4.1(5), the stiffness of the concrete, composite and masonry primary seismic members should be evaluated considering the effects of cracking. For concrete members and composite steel-concrete members, this effect may be considered by adopting the stiffness corresponding to the initiation of yielding of the reinforcement, considering mean values of material properties and the presence of axial forces.

(3) Unless a more accurate analysis of the cracked members is performed, the elastic flexural and shear stiffness properties of concrete, composite and masonry members according to (2) may be taken to be equal to one-half of the corresponding stiffness of the uncracked members or to other fractions, if any, of the corresponding stiffness of the uncracked members defined in Clauses 10, 11, 12, 13, 14 or 15.

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(4) According to prEN 1998-1-1:2022, 6.2.1(6), infill walls which contribute significantly to the lateral stiffness and resistance of the building should be considered.

NOTE See 7.4 for masonry infills of concrete, steel or composite frames.

(5) For non-linear analyses, the model should comply with prEN 1998-1-1:2022, 6.2.1 to 6.2.3.

(6) When appropriate for the application of the relevant provisions of this standard, a floor or a roof may be considered as being rigid in-plane in a given direction if it satisfies related rules per material and structural types in Clauses 10 to 15.

NOTE These rules are, in particular, detailed in 10.12, 10.14.2, 10.14.5, 11.17, 13.5(4), 13.5(5), 14.4, 14.5.2.3 and 15.13.

5.2 Minimum design eccentricity in buildings

(1) In application of prEN 1998-1-1:2022, 6.1(10), design effects of the seismic action should consider a minimal torsional effect about the vertical axis.

(2) In order to comply with (1), a minimum eccentricity, measured at right angle to the considered direction *i* of the seismic action, should be calculated at every storey *j* using Formula (5.2) and should be taken into account if it exceeds the natural eccentricity $e_{0,i,j}$.

$$e_{\mathrm{min,i,j}} = 0{,}05\, L_{\mathrm{i,j}} \tag{5.2}$$

where $L_{\mathrm{i,j}}$ is the width of the floors at the considered level *j*, measured at right angle to the direction *i* of the seismic action considered.

NOTE 1 The purpose is to cover uncertainties in cases where the natural eccentricity is very low.

NOTE 2 Annex B gives procedures for the calculation of the natural eccentricity.

(3) When the force-based approach is used, the effects at any storey *j* of mass eccentricity perpendicular to direction *i* may be calculated as resulting from a torsional moment $M_{\mathrm{i,j}}$, which acts around the building vertical axis, given by Formula (5.3).

$$M_{\mathrm{i,j}} = e_{\mathrm{i,j}} \times F_{\mathrm{i,j}} \tag{5.3}$$

where

$F_{\mathrm{i,j}}$ is the horizontal force acting on the considered storey *j* in direction *i*;

$e_{\mathrm{i,j}}$ is the maximum value of $e_{\mathrm{min},i,j}$ and $e_{0,i,j}$ when using planar models, or is equal to $e_{\mathrm{min,i,j}} - e_{0,\mathrm{i,j}}$ when using 3D models that encompass natural eccentricity for a building where $e_{\mathrm{min,i,j}}$ is greater than $e_{0,\mathrm{i,j}}$.

(4) When the non-linear static analysis is used, (1) may be considered as satisfied if, in 5.3.5.3, calculations of d_{et} and $d_{\mathrm{et,j}}$ comply with (2) and (3).

(5) When the response-history analysis is used, (1) may be considered as satisfied if the storey mass is modelled in such a way that the eccentricity of the centre of mass of the storey is at least equal to $e_{\mathrm{min,i,j}}$ in both horizontal directions.

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